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UPLIFT PRESSURES IN CONCRETE DAMS

By Kenneth B. Keener, M. ASCE

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UPLIFT PRESSURES IN CONCRETE DAMS

BY KENNETH B. KEENER,[1] M. ASCE

SYNOPSIS

Measurements of uplift pressures in concrete dams constructed by the Bureau of Reclamation (USBR) of the United States Department of the Interior have been made continuously since 1926. The justification for providing methods for, and making, such measurements and the keeping of complete records are discussed. A description of the installation and location of equipment for observing foundation uplift is given. The frequency of the readings, the over-all results obtained, and the limitations of the procedure are portrayed. A case history is included for Hoover Dam in Arizona-Nevada. A brief recounting is given of the small amount of experimental work done to determine the uplift within the concrete. The methods, together with examples, of relieving excessive foundation uplift pressures are cited. Finally, a short recital is made of the uplift assumptions used in the design of concrete dams.

GENERAL

In the design of transverse cross sections of concrete gravity dams, in addition to resistance to overturning, at least two safety factors—f_s, a sliding factor, and f_τ, a shear-friction factor[2]—are considered of importance:

$$f_s = \frac{H}{W - U} \qquad (1)$$

and

$$f_\tau = \frac{(W - U)\,\phi + A_h\,\tau}{H} \qquad (2)$$

in which H is a horizontal force; W is the weight; U is the uplift; ϕ is the coefficient of internal friction; A_h is the horizontal area; and τ is the unit shear resistance.

NOTE.—Written comments are invited for publication; the last discussion should be submitted by December 1, 1950.

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[2] "Stability of Straight Concrete Gravity Dams," by D. C. Henny, *Transactions*, ASCE, Vol. 99, 1934, p. 1041.

It is considered good and safe practice to keep the sliding factor f_s well below unity; and the shear-friction factor f_r should be five or greater. Noting the position of U, the "uplift," in Eqs. 1 and 2, one can readily determine the effect of that item in influencing the final cross section of the dam. If the assumed uplift is higher than necessary, it will be reflected in an unduly large cross section and a more costly dam. Conversely, if the assumed uplift is too low, undesirable encroachment will be made on the normally used values of the factors of safety.

Investigations of uplift pressures in dams completed by the USBR have consisted of procedures for making measurements along the contact between the concrete and the foundation rock, along construction joints in the mass concrete, and in the concrete between construction joints. Recently, some testing has been done in the USBR laboratory to determine the effect of uplift pressure on the shearing strength of concrete.

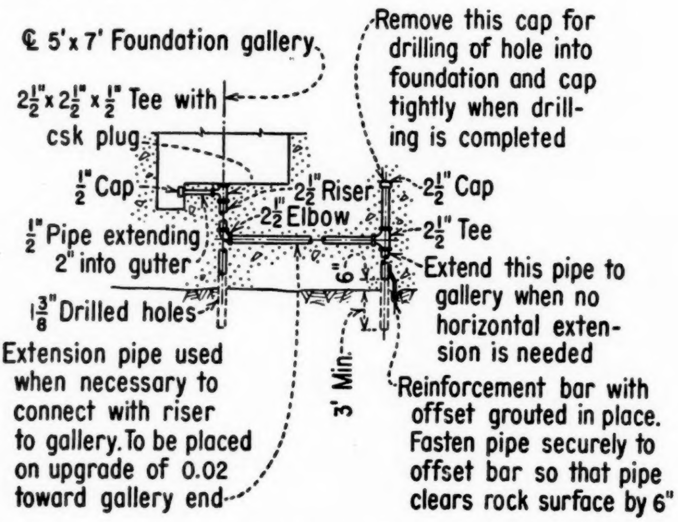


FIG. 1.—TYPICAL HOLE AND PIPE DETAILS, HUNGRY HORSE DAM IN MONTANA

The purpose of the investigations was twofold: (1) To determine what uplift assumptions should be made in the future in designing concrete dams; and (2) to observe regularly the uplift conditions at completed dams so that if uplift forces dangerously exceed the design assumptions, drainage or grouting, or both, may be accomplished to relieve such a condition.

INSTALLATIONS FOR OBSERVATION OF FOUNDATION UPLIFT

Beginning in about 1925 at American Falls Dam in Idaho, systems of vertical wells have been placed across the contacts of the bases of concrete dams with the foundation rock. The number of holes placed to date (1950) is 360 at fifteen dams, an average of 24 holes for each dam. The maximum number at any one dam was 60 at Marshall Ford in Texas, and the minimum was 6 at Olympus in Colorado. Grand Coulee in Washington has 38 holes and Shasta

in California, 53 holes. At Hungry Horse in Montana 35 holes are being drilled and there will be 24 holes at Canyon Ferry in Montana.

The layout of foundation uplift pressure pipes at Hungry Horse Dam is used for purposes of illustration of all installations. Typical hole and pipe details are shown in Fig. 1. The holes, drilled approximately 3 ft into foundation rock and having a minimum diameter of $1\frac{3}{8}$ in., are not drilled until after the $2\frac{1}{2}$-in. standard black iron riser pipe has been embedded in concrete and until foundation grouting in the surrounding area has been accomplished. Before placing concrete the lower end of the riser pipe is anchored about 6 in. above the foundation rock, by tack welding it to an offset reinforcement bar grouted in place in the foundation. Uplift pressure pipes not located directly under foundation galleries are extended to them by horizontal and vertical piping, thus permitting all uplift pressure measurements to be made within the galleries. A $2\frac{1}{2}$-in. by $2\frac{1}{2}$-in. by $\frac{1}{2}$-in. tee with a $2\frac{1}{2}$-in. countersunk (csk) plug is installed

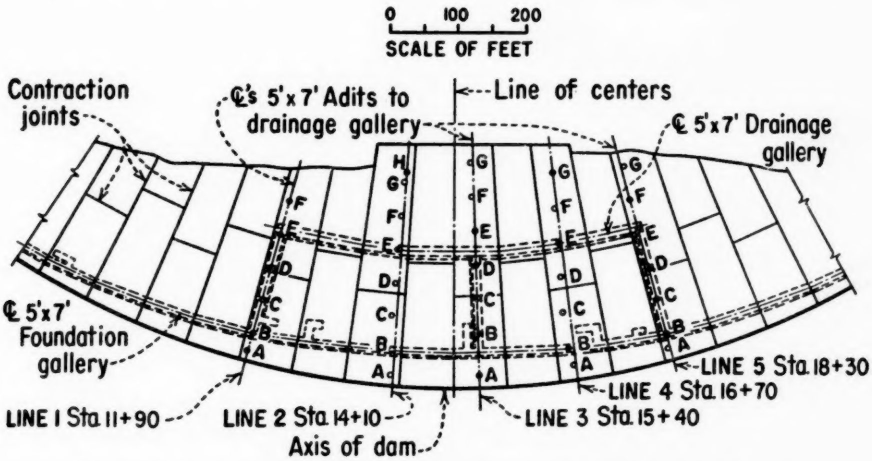


FIG. 2.—PLAN OF UPLIFT MEASUREMENT SYSTEM, HUNGRY HORSE DAM IN MONTANA

at the top of each riser pipe immediately below the gallery floor. A $\frac{1}{2}$-in. pipe extends horizontally from the tee to the gallery gutter where it is capped.

A plan of the uplift measurement system is shown in Fig. 2. The dashed lines show the location of the foundation galleries. The lines of uplift pipe are designated by numbers and dam axis stationing, and each drilled hole within a line is designated by a letter of the alphabet. The spacing between lines is determined partly by a study of the shape of the axis profile and partly by the geological characteristics of the foundation. Should all these be relatively uniform, nearly equal spacing is adopted. The spacings in this actual illustration may be noted as 220 ft, 130 ft, 130 ft, and 160 ft. The spacings between holes within a line are made nearly equal. The upstream and downstream holes are drilled within the range of from 10 ft to 50 ft from the finished faces of the dam at the foundation-concrete contacts. Naturally, the general purpose in locating uplift pressure holes is to cover the lowest areas of the foundation as well

as those of possible geological weakness where the uplift pressures would be expected to be greatest. For instance, Hole H, Fig. 2, was located within a large fault area. The profile and plan of line 2 at Station 14+10, in Fig. 3, show the locations of the holes and piping in relation to other features of the lower part of the dam.

When the reservoir water surface back of a dam reaches an appreciable height, observers begin recording uplift pressures. Initially, the plug is re-

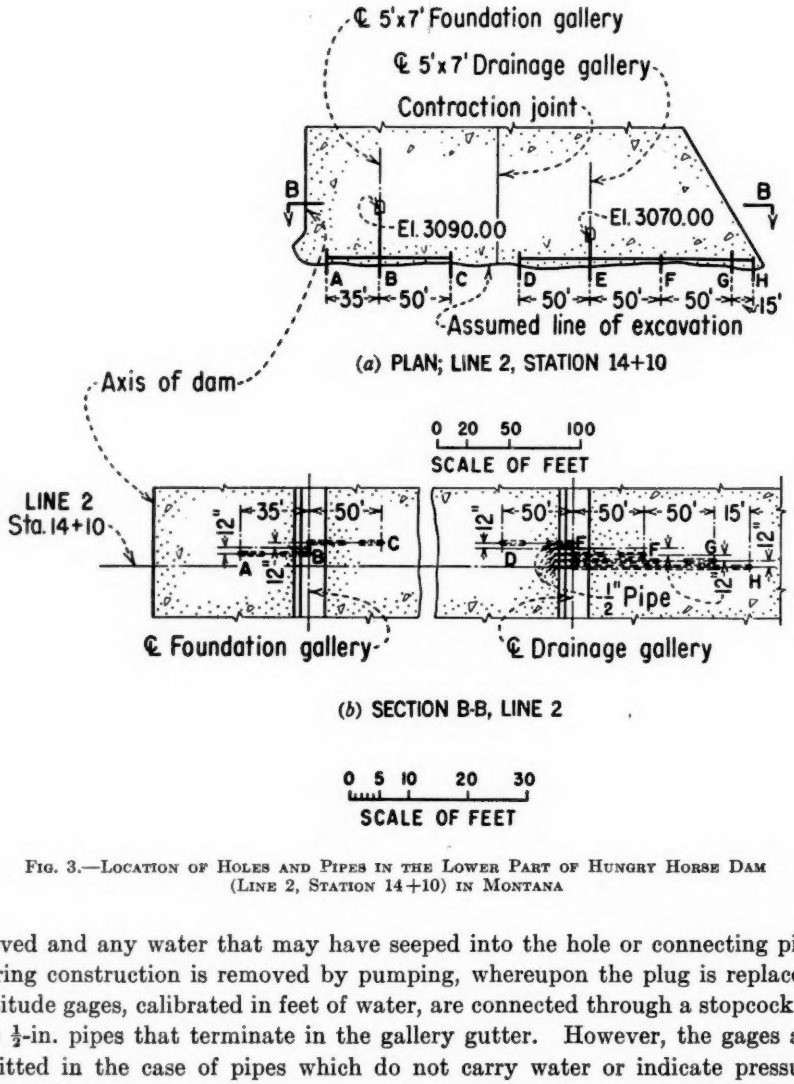


Fig. 3.—Location of Holes and Pipes in the Lower Part of Hungry Horse Dam (Line 2, Station 14+10) in Montana

moved and any water that may have seeped into the hole or connecting pipe during construction is removed by pumping, whereupon the plug is replaced. Altitude gages, calibrated in feet of water, are connected through a stopcock to the ½-in. pipes that terminate in the gallery gutter. However, the gages are omitted in the case of pipes which do not carry water or indicate pressure several days after the pumping out operation. Where the water surface in such

pipes is accessible, its elevation is determined by suspending a sounding bell on a graduated tape.

The first set of readings is made after the gages (Fig. 4) have been connected and the stopcocks have been open for about two weeks. The readings are recorded on a form such as that shown in Fig. 5. Note that both the reservoir water surface and the tailwater elevations are recorded. Three columns of the form are for records of pipes not discharging and four columns for pipe which, by reason of sufficient pressure, can discharge through the ½-in. outlet pipe connection. The elevations or equivalent elevations of the water surface in the

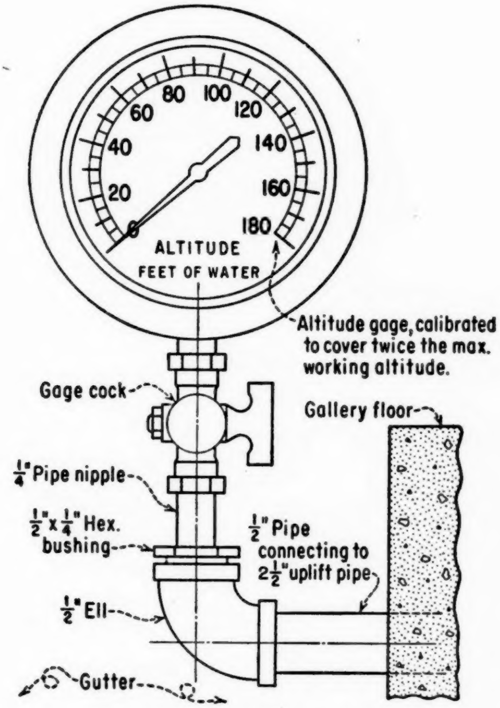


Fig. 4.—Typical Altitude Gage Installation

pipes are recorded. Readings of the pressure gages having been made, the stopcocks are closed, the gages are removed, the stopcocks are opened, and measurement of the discharge is made in gallons, in cubic centimeters, or even in drops per minute. The discharges are recorded in the last column of the form. After all measurements have been made and recorded, the gages are replaced and the stopcocks are left in the open position until the next set of readings is taken. Usually, readings are made semimonthly for the first six months of reservoir operation, after which they are made monthly only.

For illustrative purposes, although it is the actual record at Friant Dam in California during 1944, Table 1 shows uplift pressures expressed as a percentage

Date 6-13-44 Observer R. Waller
Time of beginning 10:40 A.M. Time of ending 11:59 A.M.
Reservoir W.S. Elevation 518.8 Tailwater Elevation 307.0

UPLIFT DATA AT BASE OF DAM

LOCATION	PIPE	PIPES NOT FLOWING			PIPES FLOWING			
		ELEV. TOP OF TEE	DIST. TO W.S. IN FEET	ELEV. OF W.S. IN PIPE	ELEV. OF PRES. GAGE	PRES. HEAD IN FEET	EQUIV. ELEV. W.S. IN PIPE	DIS-CHARGE G.P.M.
LINE 1 STA. 13+69.5	A	310.0	0	310.0	310.5			
	B	307.3			307.8	3.0	310.8	264*
	C	304.3			304.8	8.0	312.8	6*
	D	301.5			302.0	11.0	313.0	6*
	E	301.5			302.0	10.0	312.0	1*
LINE 2 STA. 16+25	A	310.0			310.5	52.0	362.5	0.04
	B	307.1			307.6	4.5	312.1	0.08
	C	303.9			304.4	6.5	310.9	230*
	D	301.5			302.0	8.5	310.5	0.10
	E	310.1	Inaccessible					
	F	329.7	19.0	310.7				
LINE 3 STA. 20+21	A	304.5	3.5	301.0				
	B	304.5			305.0	10.0	315.0	270*
	C	301.5			302.0	13.0	315.0	162*
	D	301.5			302.0	7.0	309.0	84*
	E	301.5			302.0	8.0	310.0	12*
	F	301.5			302.0	7.0	309.0	10.8*
LINE 4 STA. 24+35	A	369.5			370.0	51.0	421.0	0.30
	B	369.3	8.8	360.5				
	C	369.1			369.6	22.0	391.6	228*
	D	369.0			369.5	22.0	391.5	228*
	E	369.0			369.5	16.0	385.5	140*

Pipes are lettered in upstream to downstream direction.
*Indicates drops per minute.

FIG. 5.—UPLIFT PRESSURE READINGS AT FRIANT DAM IN CALIFORNIA (TABLE 1)

TABLE 1.—UPLIFT PRESSURES AT FRIANT DAM IN

Date of record	Head, in feet, lines 1, 2, 3	LINE 1					LINE 2					
		A	B	C	D	E[a]	A	B	C	D	E	F
March 11	120.30	3.74	3.99	4.82	4.99	0.42	26.18	....	4.07	3.33	....	....
March 15	126.90	2.68	2.92	3.70	3.86	−2.02	25.14	0.79	3.39	1.89	....[b]	2.44
April 14	132.30	1.89	2.12	3.25	2.65	1.89	23.43	1.97	2.57	1.51	....[b]	1.89
May 10	158.10	1.77	1.64	2.91	3.04	1.77	22.33	1.52	1.71	1.45	....[b]	2.09
June 13	211.80	1.42	1.79	2.74	2.83	2.36	26.20	2.41	1.84	1.65	....[b]	1.75
July 29	209.20	1.24	1.63	2.58	2.68	3.15	23.47	2.96	1.67	1.72	1.29	1.43
August 25	185.80	1.61	2.05	2.85	2.96	2.69	23.41	2.74	1.56	1.61	....[b]	2.31
September 23	163.40	1.77	1.65	2.88	3.00	3.00	23.50	2.75	2.02	1.77	....[b]	1.90
October 28	147.70	2.64	2.84	3.86	3.66	5.69	27.01	2.71	1.90	1.96	....[b]	2.44
November 28	170.30	2.86	2.99	3.88	3.99	5.75	24.54	3.76	2.76	1.94	....[b]	2.64
December 30	195.60	2.51	2.66	3.68	3.53	5.57	25.77	3.32	2.20	1.74	....[b]	2.51
Maximum	211.80	3.74	3.99	4.82	4.99	5.75	27.01	3.76	4.07	3.33	....	2.64
Minimum	120.30	1.24	1.63	2.58	2.65	0.42	22.33	1.52	1.56	1.45	....	1.89
Average	165.58	2.19	2.39	3.38	3.38	3.54	24.63	2.68	2.34	1.87	....	2.14

[a] Negative values of uplift pressure are shown only to indicate that pressures are below tailwater elevation

of head. The uplift pressure at each hole in the foundation is reduced to a percentage of total head acting on the dam at the line of pipes within which the hole is located. Total head is taken as the difference between the reservoir water surface and the tailwater surface elevations; or, if there is no tailwater at the line of holes, the total head is the difference between the reservoir water

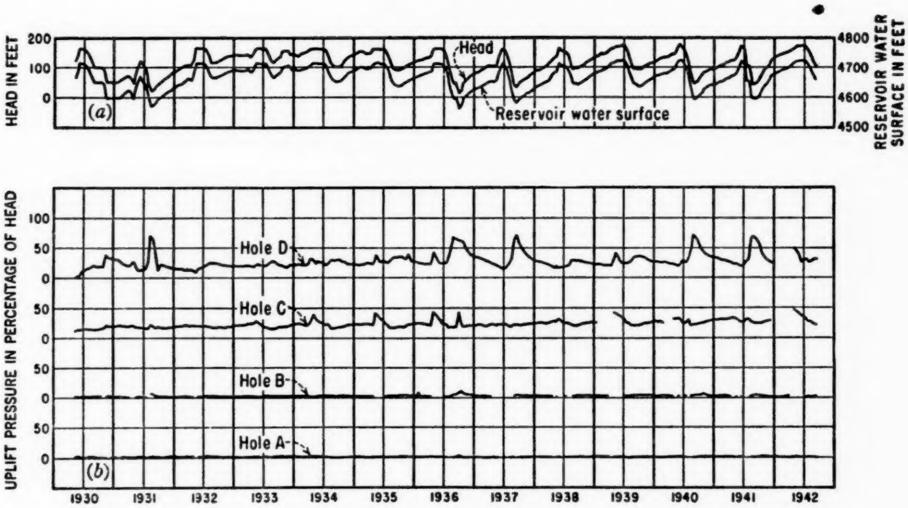


FIG. 6.—UPLIFT PRESSURES AT THE BASE; LINE 2, GIBSON DAM IN MONTANA

surface elevation and the elevation of the concrete-rock contact. The latter elevation is taken as the average of all concrete-rock contact elevations as determined from the uplift holes within a line of holes. At the bottom of Table 1, the maximum, minimum, and average values of the uplift pressures, in percentage of head for each hole for the year, are noted. Graphs are prepared and

CALIFORNIA EXPRESSED AS PERCENTAGES OF HEAD (1944)

LINE 3						LINE 4					Head, in feet, line 4	Date of record
A[a]	B	C	D[a]	E	F[a]	A	B[a]	C	D	E		
....	....	....	....	....	....	....	....	....	....	....	....	March 11
−6.38	3.47	3.47	3.07	0.71	−0.08	30.57	−1.32	11.47	20.26	19.58	72.74	March 15
−6.35	2.65	2.65	1.89	0.38	0	39.52	−1.21	10.55	21.18	19.28	79.04	April 14
−4.74	3.04	3.04	2.09	0.82	0.19	37.54	−0.82	19.93	19.84	17.93	104.54	May 10
−2.83	3.78	3.78	0.94	1.42	0.94	38.12	−0.16	19.51	19.45	15.65	158.04	June 13
−1.39	3.15	3.39	0.29	1.24	0.76	39.30	0.47	19.79	19.08	15.23	155.84	July 29
4.04	3.23	3.23	0	1.08	0.54	40.32	0.64	19.95	18.74	15.33	132.04	August 25
2.39	3.00	2.69	−0.67	0.55	0.24	42.14	0.95	19.90	17.99	16.17	109.74	September 23
3.32	3.32	3.16	0.27	0.95	0.61	42.71	1.33	19.60	17.99	16.92	93.04	October 28
3.70	3.70	3.70	1.64	1.35	1.06	39.43	0.99	17.29	15.90	14.15	114.74	November 28
1.48	3.78	3.78	0.97	1.74	1.23	36.62	1.10	17.75	13.39	13.39	139.94	December 30
4.04	3.78	3.78	3.07	1.74	1.23	42.71	1.33	19.95	21.18	19.58	158.04	Maximum
1.48	2.65	2.65	0.94	0.71	0.54	30.57	0.47	10.55	11.33	13.39	72.74	Minimum
2.99	3.31	3.29	1.77	1.16	0.86	38.63	0.91	17.57	17.72	16.36	115.97	Average

and are omitted in computations for average values for year. [b] Pipe inaccessible for measurement.

kept up to date showing a running record of the uplift pressure in percentage of head for each hole. Fig. 6 shows such graphs for the holes in line 2 at Gibson Dam in Montana. Records of the hydraulic heads and of the elevations of the reservoir water surface are shown in Fig. 6(*a*). The entire history of recorded uplift pressures for a dam may be reviewed on such graphs.

Fig. 7 shows two charts, typical of what may be done in analyzing uplift pressure values. These were prepared from uplift pressure readings at American Falls Dam from 1926 to 1939. Fig. 7(*a*) shows the point-to-point variation across the foundation of the dam for the average uplift pressures at each hole within a line of holes for the period of record. Each line of holes at the dam is included and the average of all lines is shown by the heavier full line. Fig. 7(*b*) is the same, except that the plotting is the result of averaging the yearly maximum uplift pressure readings at each hole regardless of the date in which the maximums were recorded. For comparative purposes, full, two-thirds, and

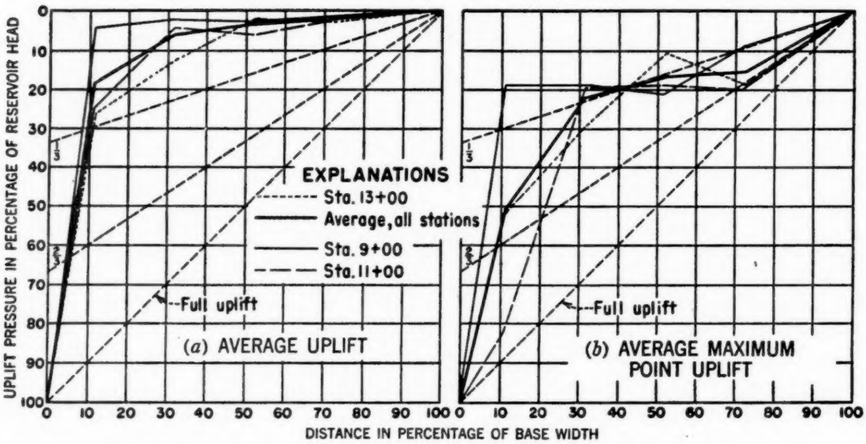
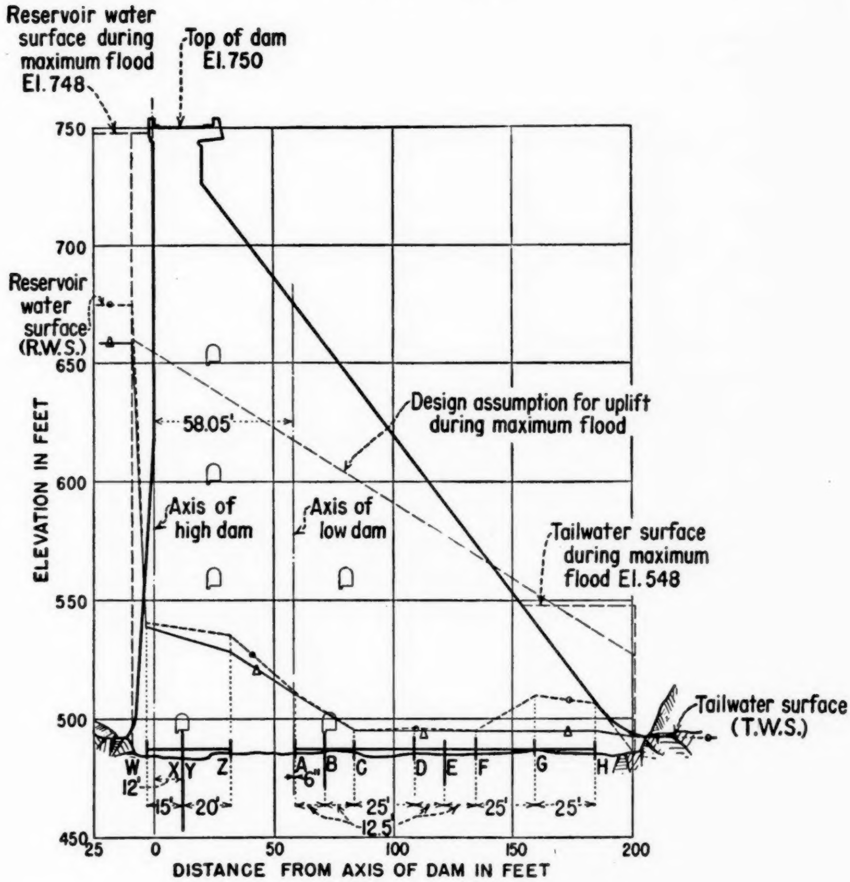


FIG. 7.—UPLIFT PRESSURE AT BASE OF DAM, 1927 TO 1939, AMERICAN FALLS DAM IN IDAHO

one-third uplift pressures varying linearly from 100% reservoir water pressure at the upstream face of the dam to zero pressure at the downstream face are shown on both charts.

Fig. 8 is an example of a sectional view along a line of uplift holes. This particular illustration is for line 3 at Marshall Ford Dam. Plottings of the uplift pressures for two different reservoir water surface elevations have been made on the section, as well as a line showing the design assumption for uplift during a period of maximum flood. In plotting the uplift pressures the results from holes Y, B, and E have not been used as they were drilled considerably beyond the conventional depth of 3 ft below the foundation in order to determine the uplift experimentally in such locations. The upper strata were cased off so that actual foundation pressures are not reflected in the observations. Pressures at holes X and Y in the same location differed in that the observations at the latter (which is about 22 ft deeper than hole X) were from 15 ft to 19 ft

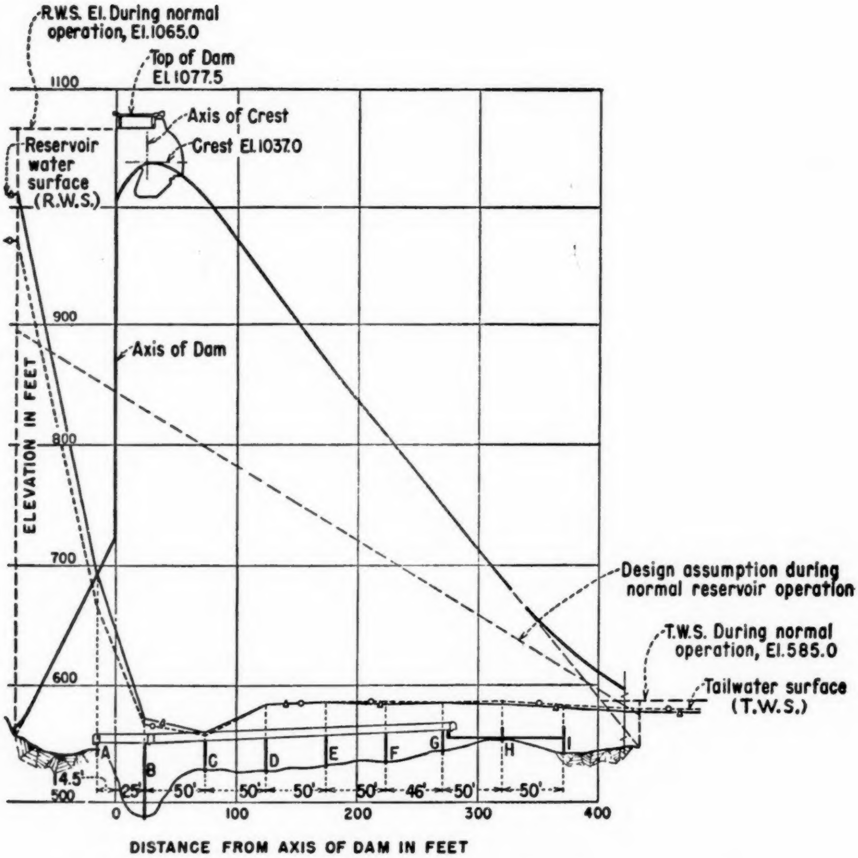


DATE	R.W.S. EL.	EL. OR EQUIVALENT EL. OF W.S. IN PIPE												T.W.S. EL.	LEGEND
		W	X	Y	Z	A	B	C	D	E	F	G	H		
JAN. 18, 1946	664.04	539	504	523	524	510	495	495	496	N.W.	495	508	505	492.20	
FEB. 20,	667.36	539	505	523	529	510	495	495	495	488	495	508	504	492.40	
MAR. 21,	669.80	541	505	524	534	510	495	495	495	N.W.	495	510	507	492.60	
APR. 23, 1946	674.89	541	510	524	536	511	495	495	496	N.W.	495	510	507	492.20	--o--
MAY 20,	677.53	543	509	524	537	510	495	495	495	N.W.	495	511	508	492.80	
JUNE 20,	674.42	542	509	524	534	511	495	495	495	N.W.	495	510	507	492.32	
JULY 22,	673.26	542	510	524	534	511	495	495	495	N.W.	495	495	495	491.92	
AUG. 20,	668.97	541	509	524	533	511	495	495	495	N.W.	495	495	495	493.16	
SEPT. 19,	662.35	539	509	524	531	—	495	495	495	N.W.	495	503	495	492.62	
OCT. 21, 1946	658.67	539	509	524	529	510	495	495	495	N.W.	495	495	495	492.60	—△—
NOV. 21,	661.24	539	509	524	529	510	495	495	495	N.W.	495	503	495	492.00	
DEC. 19, 1946	664.38	537	509	524	524	510	495	495	495	N.W.	495	503	495	492.54	

N.W. Indicates no water at measurable depth.

LINE 3 - STA. 40 + 23.5

FIG. 8.—UPLIFT PRESSURES AT FOUNDATION; LINE 3 (STATION 40+23.5), MARSHALL FORD DAM IN TEXAS (MEASUREMENTS DURING 1946)



DATE	R.W.S. EL.	EL. OR EQUIVALENT EL. OF W.S. IN PIPE									T.W.S. EL.	LEGEND
		A	B	C	D	E	F	G	H	I		
JAN. 22, 1947	964.0	668	569	559	583	586	585	586	586	583	580.9	
FEB. 19,	974.1	673	569	559	585	587	586	587	586	583	577.8	
MAR. 18,	995.2	680	571	560	587	587	586	586	586	582	577.8	
APR. 16, 1947	1010.6	689	572	560	584	586	585	586	585	582	577.8	
MAY 28,	1006.1	690	572	560	584	587	586	586	586	583	581.4	
JUNE 25,	1006.4	689	572	560	584	587	587	587	586	584	581.8	
JULY 23,	996.4	680	571	560	584	586	585	586	586	583	582.1	
AUG. 20,	984.9	673	570	559	583	586	586	586	586	583	581.7	
SEPT. 17,	976.1	668	569	559	582	585	585	585	585	583	579.8	
OCT. 15, 1947	972.3	665	568	559	584	586	586	586	586	583	579.7	
NOV. 26,	973.1	664	569	559	583	586	583	585	585	582	579.3	
DEC. 23, 1947	972.8	664	569	559	583	585	585	586	585	582	578.2	

LINE 4 - STA. 26 + 06.75 ON ℄ OF TRANSVERSE GALLERY

FIG. 9.—UPLIFT PRESSURES AT FOUNDATION; LINE 4 (STATION 26+06.75) ON CENTER LINE OF TRANSVERSE GALLERY, SHASTA DAM IN CALIFORNIA (MEASUREMENTS DURING 1947)

greater than those from hole X. Sectional views and tabulations are prepared for each line of holes for each dam for each year, the number of lines plotted varying according to the variations indicated by the tabulation and other considerations. Frequently, only the maximum and minimum elevations of uplift pressure are plotted. Fig. 9 is another example of a sectional view of measurements during 1947 for line 4 at Shasta Dam. The uplift measurements are quite small when compared to the design assumption during normal reservoir operation. The illustrative figures show how closely the uplift pressures

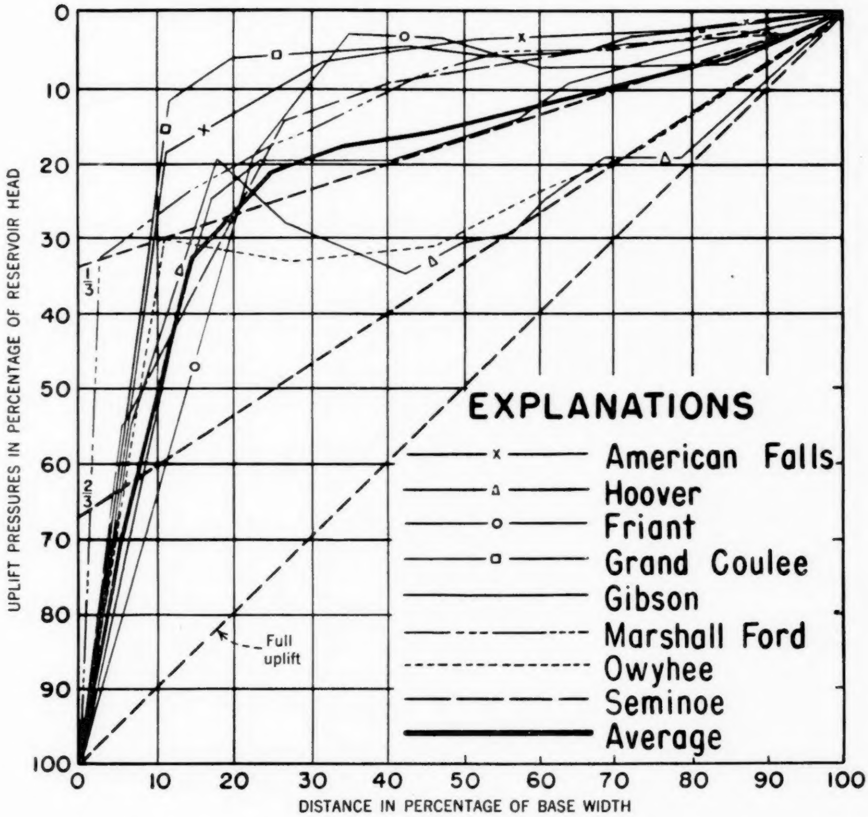


Fig. 10.—Average Uplift Pressure at the Base of Several Dams

can be watched in order to detect any unusual conditions which may develop and which deserve remedial attention.

Fig. 10 presents a chart prepared similarly to the charts in Fig. 7 but which gives the average foundation uplift pressure, to the time of preparation of the chart, for eight dams. The average of all lines is represented by the heavy full line. That line is indicative of the general trend of foundation uplift pressures transversely across concrete dams, the foundations of which are provided with

drains and grout curtains. It might be used in considering design assumptions for future concrete dams. It is interesting to note that the measured pressures are generally less than a two-thirds uplift pressure assumption, but that, as might be expected, the higher pressures are near the upstream face of a dam.

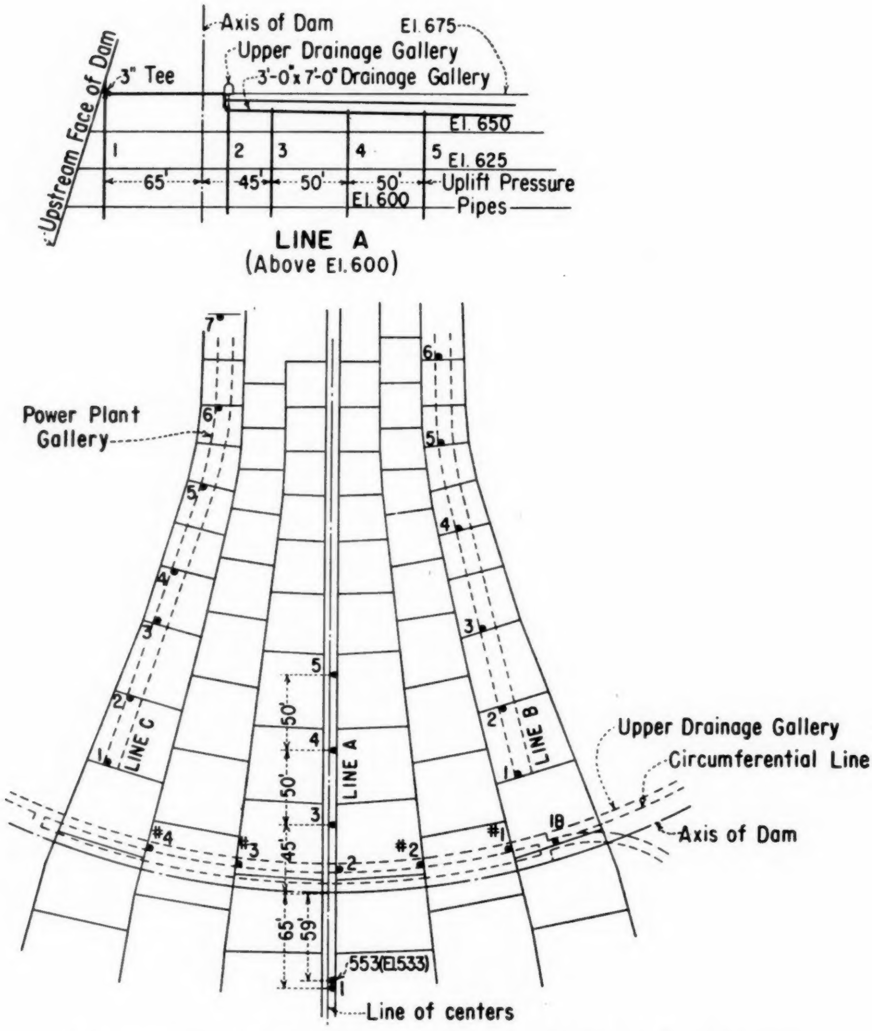


FIG. 11.—LOCATION OF UPLIFT PRESSURE HOLES, HOOVER DAM IN ARIZONA-NEVADA

The method described for observing foundation uplift pressure has some limitations. Although widely dispersed, the holes cover only a small percentage of the foundation area. Other than those holes which have direct vertical access to galleries, it is difficult, if not impossible, to clean out holes that have

become clogged. There has been some difficulty in keeping the gages in perfect operating condition, because the moving parts tend to corrode in the damp atmosphere of the galleries. They should be recalibrated every few months. As mentioned previously, after the pressure observations have been made, it has been the practice to remove the altitude gages and measure the discharge from the stopcocks. Naturally, the pressure dissipates for the time being and there is always the possibility that it may not be fully restored before the next regular reading. Accordingly, the practice of estimating the amount of water dis-

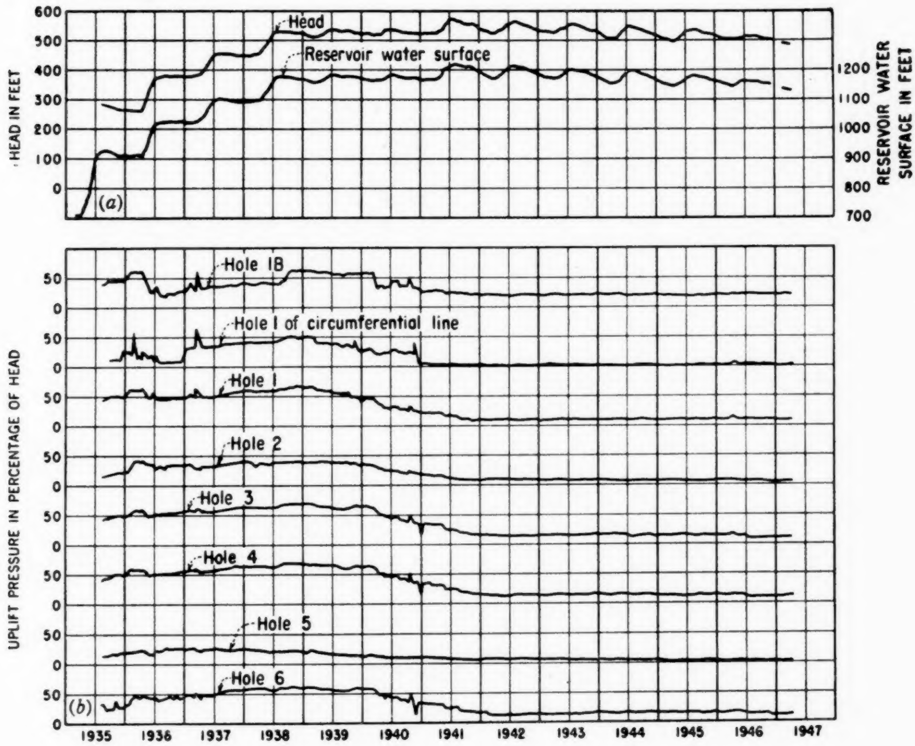
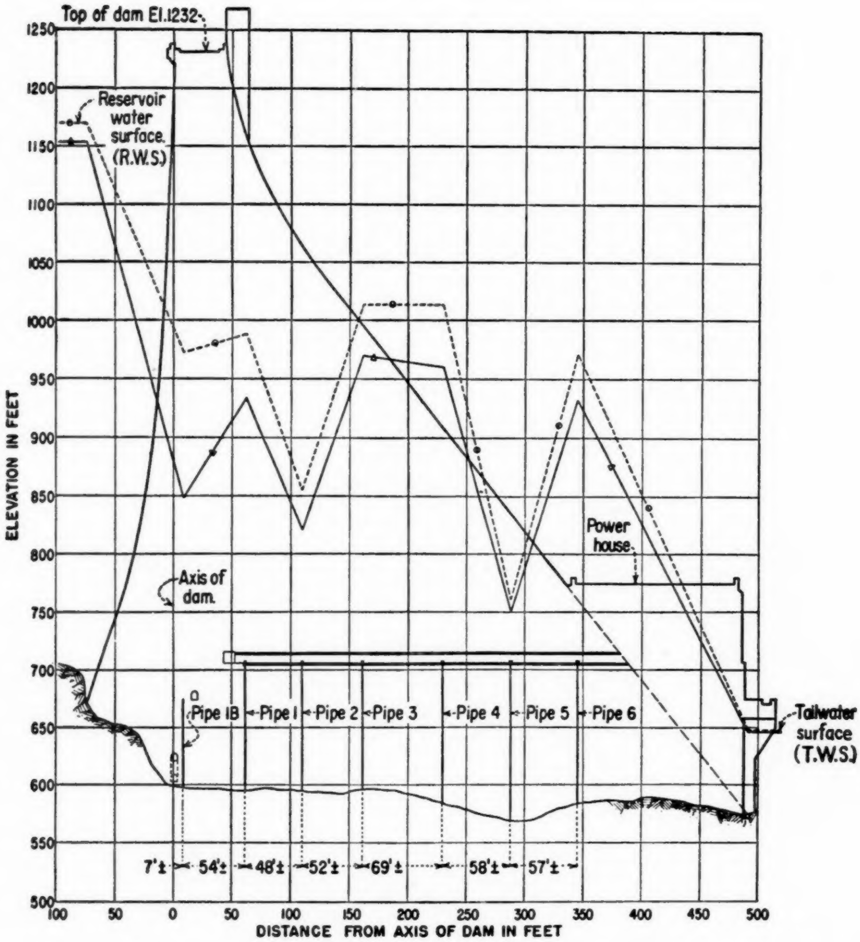


Fig. 12.—Uplift Pressure at Base; Line B, Hoover Dam in Arizona-Nevada

charged has been discontinued. It is doubtful if the revised procedure will make any material difference in the observations except in isolated cases.

Case History at Hoover Dam

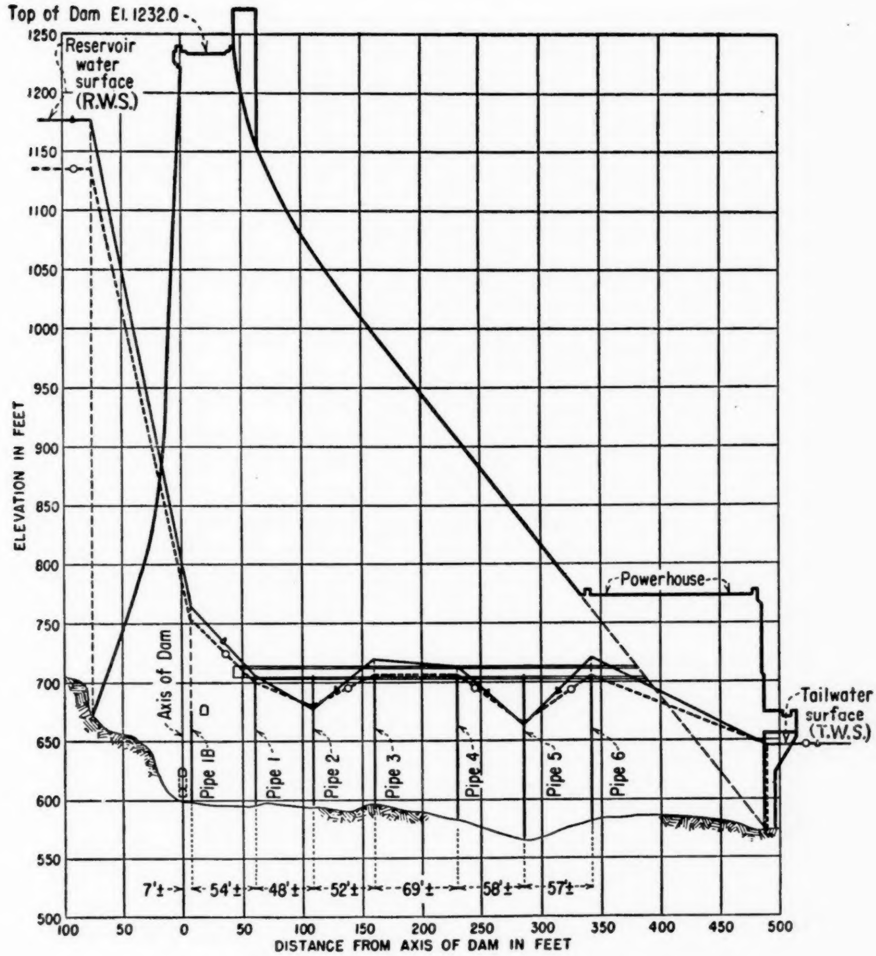
The essential case history of foundation uplift pressures at Hoover Dam is of interest. Fig. 11 shows the location of the 24 foundation uplift pressure holes—three radial lines and one circumferential line. Uplift pressures were first recorded in August, 1935, and have been continued to date. The higher pressures occurred during the first five years. Fig. 12, a running record for the holes in line B, reveals an increase in pressures until 1939 when remedial measures



DATE	R.W.S. EL.	EL. OR EQUIVALENT EL. OF W.S. IN PIPE							T.W.S. EL.	LEGEND
		1B	1	2	3	4	5	6		
JAN. 14, 1938	1095.33	822.05	909.78	825.67	933.92	938.00	757.06	906.18	645.47	
FEB. 15,	1094.65	821.05	904.78	821.67	930.92	935.00	750.06	909.18	645.60	
MAR. 15,	1099.15	814.05	909.78	818.67	934.92	937.00	750.06	911.18	645.05	
APR. 15,	1100.75	836.05	909.78	819.67	938.92	940.00	749.06	915.18	646.75	
MAY 13,	1119.90	838.05	918.78	819.67	944.92	948.00	750.06	922.18	645.55	
JUNE 15, 1938	1152.93	848.05	933.78	819.67	970.92	960.00	750.06	932.18	645.70	—o—
JULY 15,	1172.59	853.05	957.78	843.67	983.92	987.00	760.06	953.18	646.06	
AUG. 15,	1172.85	859.05	974.78	856.67	1000.92	999.00	761.06	966.18	645.95	
SEPT. 15,	1173.03	890.05	978.78	857.67	1006.92	1007.00	763.06	970.18	646.40	
OCT. 14,	1172.81	970.05	987.78	855.67	1008.92	1013.00	766.06	977.18	647.00	
NOV. 14, 1938	1169.19	972.05	988.78	854.67	1013.92	1014.00	761.06	971.18	647.12	---o---
DEC. 16, 1938	1168.37	972.05	987.78	854.67	1007.92	1010.00	755.06	966.18	646.84	

LINE B ALONG 10-FOOT GALLERY FROM ELEVATOR TO POWER HOUSE-NEVADA SIDE

FIG. 13.—UPLIFT PRESSURES AT FOUNDATION; LINE B, HOOVER DAM IN ARIZONA-NEVADA (MEASUREMENTS DURING 1938)



DATE	R.W.S. EL.	EL. OR EQUIVALENT EL. OF W.S. IN PIPE							T.W.S. EL.	LEGEND
		1B	1	2	3	4	5	6		
JAN. 15, 1947	1146.22	754	700	682	707	706	666	707	647.00	
FEB. 14,	1140.40	754	701	682	707	707	665	707	647.00	
MAR. 14,	1136.08	755	700	681	707	707	665	707	647.00	
APR. 15, 1947	1135.12	755	700	681	707	707	665	707	647.00	--○--
MAY 15,	1141.03	755	700	681	707	707	665	707	647.00	
JUNE 16,	1160.52	758	702	680	707	707	664	707	647.00	
JULY 15,	1175.65	762	704	680	707	707	664	707	647.00	
AUG. 15,	1177.23	766	706	680	719	714	665	721	647.00	
SEPT. 15,	1179.40	766	707	680	720	714	665	722	647.00	
OCT. 15,	1176.78	767	707	680	720	714	664	722	647.00	
NOV. 14, 1947	1177.01	766	707	679	720	714	664	722	647.00	—△—
DEC. 15, 1947	1173.55	766	707	679	720	714	664	722	647.00	

LINE B ALONG 10-FOOT GALLERY FROM ELEVATOR TO POWERHOUSE-NEVADA SIDE

FIG. 14.—UPLIFT PRESSURES AT FOUNDATION; LINE B, HOOVER DAM IN ARIZONA-NEVADA (MEASUREMENTS DURING 1947)

were undertaken by drilling more drain holes and by further foundation grouting. Measurements and experimental observations made during 1936 and 1937 indicated that, when the drains were closed intentionally, the uplift pressures increased and that, when they were opened, there was some decrease in pressure. After the drilling of additional drain holes, the pressures gradually decreased and the decrease was more rapid after supplemental grouting of the foundation. By 1942 the pressures had been reduced to nominal amounts that have remained generally constant since about 1942. Fig. 13 shows the uplift pressure measurements on line B during 1938; and, in contrast, Fig. 14 shows the same during 1947.

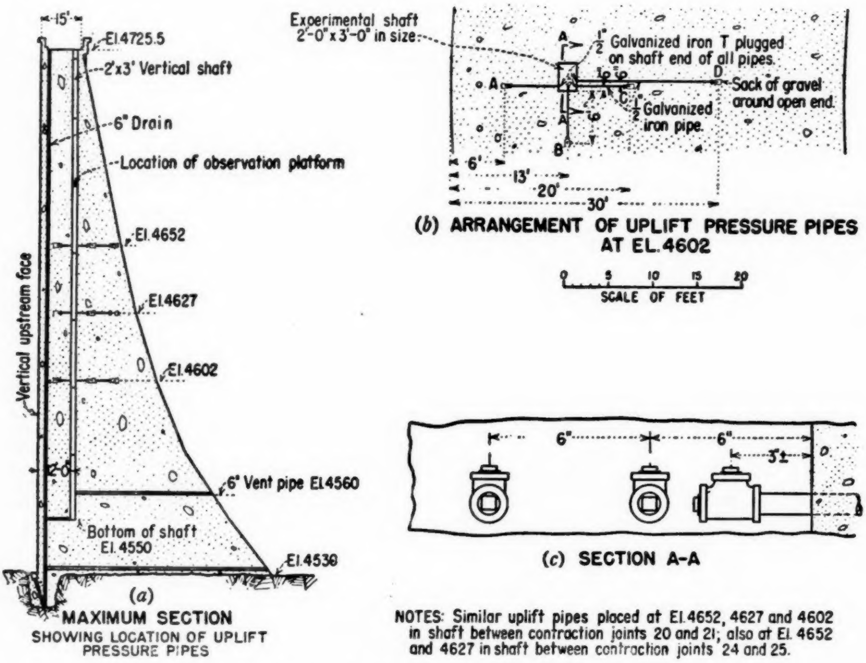


FIG. 15.—PIPES FOR MEASURING UPLIFTS IN CONCRETE, GIBSON DAM IN MONTANA

UPLIFT WITHIN THE CONCRETE

Equipment to determine the uplift within the mass concrete at horizontal construction joints has been installed in Gibson, Owyhee (in Oregon), and Hoover dams. Fig. 15 shows the installation at Gibson Dam. Sacks of gravel were embedded in the concrete around the open ends of ½-in.-diameter pipe, which led to a small vertical shaft where measurements of seepage or uplift could be made. There were five systems or sets of uplift pipes, identical with the arrangement shown in Fig. 15(*b*), installed at various locations throughout the dam. Frequent observations were made on these uplift units beginning in May, 1930. There was no indication of uplift pressure in any of the units. During June and August, 1930, a slight amount of water appeared in the pipes

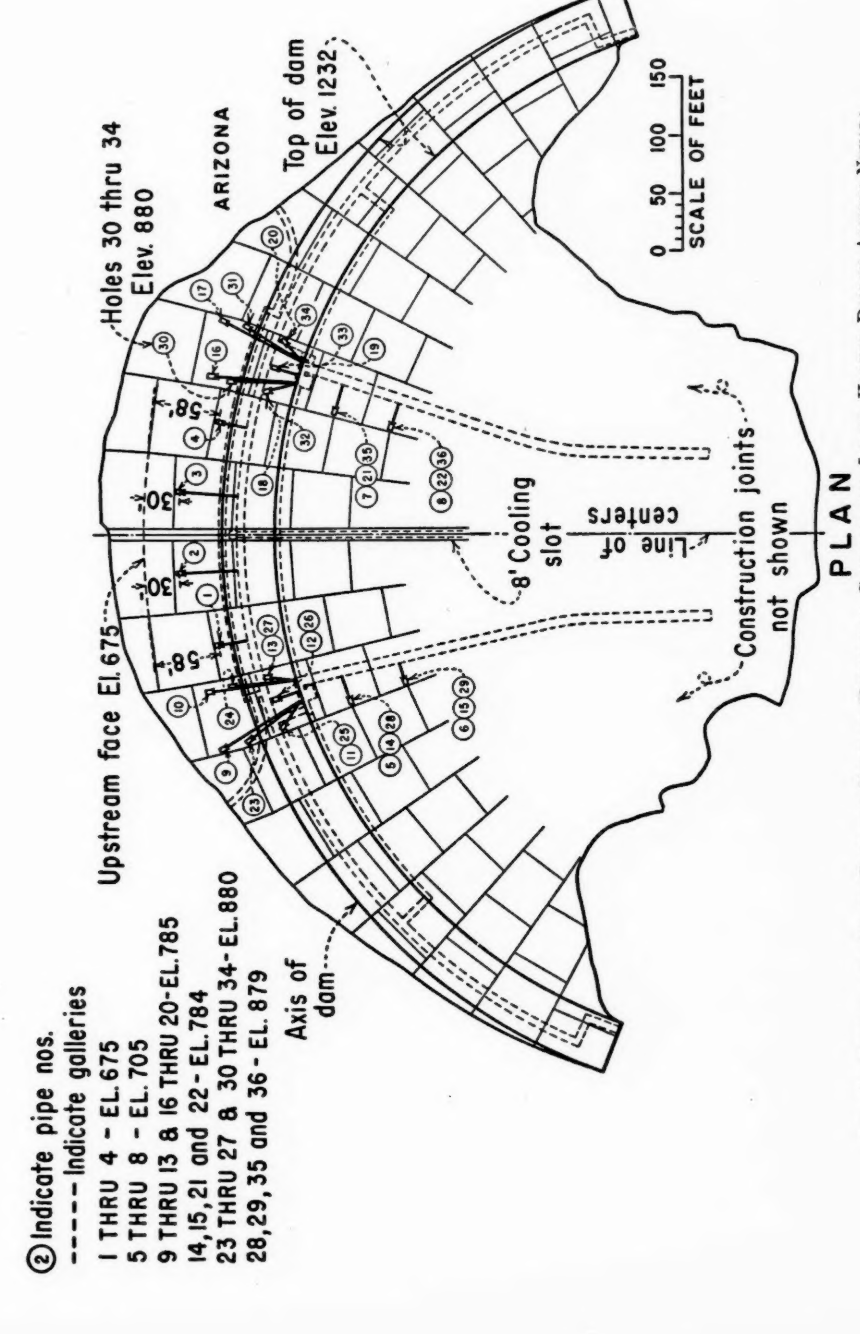


Fig. 16.—Location of Uplift Pressure Pipes at Horizontal Construction Joints, Hoover Dam in Arizona-Nevada

from units A and B at El. 4602, and those pipes were noted as damp in March and April, 1931. At two other units at El. 4627 and El. 4652, a few drops of water were recorded in June, 1930. Otherwise, all units were dry and, after monthly observations had been made for eleven years with no signs of moisture in any of the units, observations were discontinued in September, 1942, except for quarterly check readings.

Similar installations were made at Owyhee Dam to the extent that it was possible to measure the uplift at twenty-four different locations throughout the mass concrete. Observations since May, 1943, have shown no uplift pressure or even any indication of moisture.

Fig. 16 shows the locations of thirty-six uplift units installed at horizontal construction joints within the mass concrete of Hoover Dam. With almost negligible exceptions, since observations began in August, 1935, no uplift or moisture has been indicated.

Some comparatively recent tests were made in the USBR laboratories of 337 specimens to determine the effective area over which uplift pressure acts. Methods and results of those tests have been described by Douglas McHenry,[3] M. ASCE. The method of determining the effective area was indirect, consisting of measuring the effect of induced pore pressure on shearing strength. Specimens were tested with and without pore pressure, and from the observed reduction in shearing strength the uplift force was found. By the Coulomb equation and by various combinations of testing specimens of the same material with different combinations of direct axial and circumferential fluid loadings, the effective area of pore pressure at failure was determined. As a result of those indirect tests (of which Mr. McHenry noted that there were many possibilities of inaccuracy) it was indicated that the pore pressure was effective over almost the entire surface of the area of failure. It is well known that many capable engineers have studied extensively the vexing problem of determining the effective area over which uplift acts. However, this paper is confined to activities of the USBR and does not include the many endeavors of others on this subject. It is possible that a more direct solution will be evolved.

Relief from Excessive Uplift

Everyone is familiar with the procedure of preparing the foundation for a dam so that uplift pressure will be alleviated as much as practicable. This involves deep curtain grouting near the axis and a row of drainage holes immediately downstream. Where the geological formations are porous, badly jointed, or otherwise comparatively permeable, there will be blanket grouting over the entire area before concrete is placed. Immediately before placing concrete, the foundation rock surface is thoroughly cleaned of all foreign matter and covered with a thin layer of mortar, from ½ in. to 1 in. thick, which is thoroughly brushed into all crevices and over the entire surface of the foundation. The mortar is the same as that of the regular concrete mixture. The mortar layer is intended to provide a tight contact or bond between the mass concrete and the foundation rock.

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[3] "The Effect of Uplift Pressure on the Shearing Strength of Concrete," by Douglas McHenry, *Report No. 48* Vol. I, International Comm. on Large Dams, Paris, France, 1948.

After a dam has been completed, observations of foundation uplift pressure sometimes indicate that the design assumptions have been exceeded to such an extent that the uplift should be reduced. In such event, access through various galleries in the dam make it fairly easy to tighten or extend the grout curtain, to clean out the drain holes by reboring, or to drill additional drain holes.

Reduction of uplift pressure at Hoover Dam by supplemental grouting and drainage has been illustrated by reference to conditions at line B: Fig. 12 shows the record before and after the supplemental work, which was completed about 1941; Fig. 13, the uplift during 1938; and Fig. 14, the same during 1947.

A program of supplemental grouting and drainage was conducted at Owyhee Dam during 1936 and 1937, about four years after completion of that structure. Although the primary purpose was to reduce seepage of water through the left abutment rock, undesirable uplift more pronounced on that part of the dam near the left abutment was reduced.

Observations on one of the smaller concrete gravity dams showed that the uplift pressure in several locations was greater than that used in the design assumption. As a first step in reducing the pressure, additional drain holes were drilled between the existing drain holes which are on 10-ft centers. That process resulted in sufficient reduction. Had it not, the grout curtain could have been tightened or extended. At another dam where uplift pressures have increased since construction beyond those which are considered to be desirable, it is hoped to accomplish satisfactory reduction merely by reaming out the existing drainage holes which give indications of having become plugged with natural deposits.

Uplift Design Assumptions

The maximum and most common uplift design assumption used by the USBR in the past for concrete gravity dams is that uplift pressure on the base varies uniformly from full-reservoir pressure at the upstream toe to tailwater elevation or zero, as the case may be, at the downstream toe and that the pressure acts over two thirds of the base area. Some of the exceptions to that common assumption were noted by Ivan E. Houk, M. ASCE, 1932.[4] For example, it was assumed for Owyhee Dam that the pressure would vary uniformly from full-reservoir head at the upstream toe to one half of that head at the line of foundation drains (located 6.5 ft downstream from the axis) and from there uniformly to tailwater head at the downstream toe, and that such pressures would act over the entire base of the dam. The common assumption of full-reservoir head varying uniformly to tailwater elevation and applied to two thirds of the base area was used in the designs of Hoover, Marshall Ford, Friant, Grand Coulee, Shasta, Keswick (in California), Kortes (in Wyoming), and Canyon Ferry dams.

Observations of uplift on existing dams not only are of value in detecting unusually high pressures, but also are of value in considering uplift design assumptions for proposed dams. Such assumptions may best be determined by the designer after judging the past performance of uplift on existing dams with similar foundations in conjunction with the definite plans for grouting and

[4] "Measurements at Existing Structures," by Ivan E. Houk, *Civil Engineering*, September, 1932, p. 578.

draining the foundation at the proposed dam. It is readily recognized that the uplift design assumption for a dam to be constructed on a porous or badly jointed rock member should be higher than for one on a tight geological formation. The possibility that uplift pressures may be unusually slow in acting and may not be effective until after a long period must be given some weight.

Considering the extraordinarily low uplift pressures so far indicated over a short period for Shasta Dam, as shown in Fig. 9, and the average uplift pressures indicated by Fig. 10 for several dams over comparatively long periods, one is faced with the question of whether the uplift assumptions of the past have, in general, been too conservative. A higher-than-necessary uplift assumption results in actual factors of safety in the completed dam which are beyond the intention of the designer. With such an assumption, the dam is made heavier than necessary to resist overturning (should overturning be a governing factor), the shear-friction factor is actually greater than five (which is considered ample), and the sliding factor is much less than the value intended for safety. In brief, an uplift assumption more in keeping with the observations that have been made would result in smaller cross sections and appreciable reduction in cost. Should there be some skepticism regarding the adequacy of an uplift design assumption, if one has confidence in the reliability of the equipment and observations, then for dams with foundation galleries there is the insurance that uplift higher than that assumed may be relieved by supplemental grouting and drainage. Thus, a first-cost saving in reducing the section of a dam might or might not be fully realized in any particular case. However, it would appear that for a number of dams there would be a resultant saving in cost.

Conclusion

An attempt has been made to give factual data on the experience of the USBR with uplift pressures on concrete dams. The greater part of such experience has consisted of the installation of equipment for, and the observation of, foundation uplift pressures. The value of those observations may be separated into two categories: First, the periodic observation of uplift pressures is an operational guide for determining the necessity of any remedial measures that should be undertaken to reduce undue pressures in existing dams; and, second, the observations may be utilized in considering uplift design assumptions for proposed dams.

Acknowledgment

This paper, in considerably more detail, was presented to the Power Division at the 1949 Fall Meeting of ASCE in Washington, D. C., at the request of the chairman of the Joint Committee on Masonry Dams.